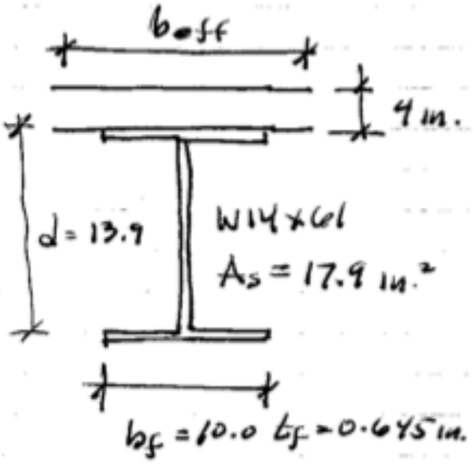
**I.** Complete the following problems from the textbook (LRFD only):

Chapter 9 – Composite Construction: 9-6, 9-15, 9-30

**9-6**




$b_{eff} = \frac{24}{4} = 6\text{ ft} = 72\text{ in.}^*$

or $= 6$ ft

DETERMINE LOCATION OF PNA

$C_c = 0.85(3)(72)(4) = 734^{k}\,*$

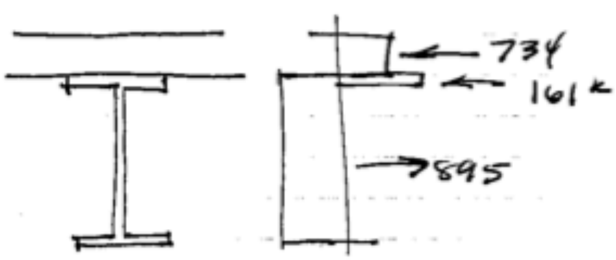
$C_s = 17.9(50) = 895^{k}$

SINCE $C_c < C_s$ PNA IS IN STEEL

$$A_{s_{comp}} = \frac{895 - 734}{2(50)} = 1.61\text{ in}^2$$

assume PNA in flange

$$x = \frac{1.61}{10.0} = 0.161 < t_f = 0.645$$




$$M_n = 895\left(\frac{13.9}{2}\right) + 734\left(\frac{4}{2}\right) - 161\left(\frac{0.161}{2}\right) = 7680\text{ in-k}$$

$$= 640\text{ ft-k}$$

a) $\phi M_n = 0.9(640) = \underline{576\text{ ft-k}}$

**9-15**

From Prob 6 W14x61

$C_c = 734^k$; $C_s = 895^k$ $b_f = 10.0$ in.

$C_q = V'_q = 600$ k * $t_f = 0.645$ in.

IN CONCRETE

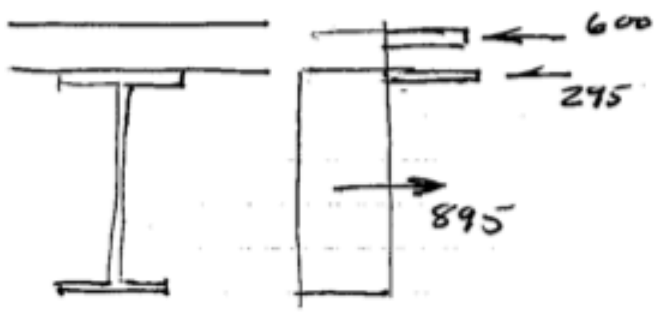
$$a = \frac{600}{0.85(3)(72)} = 3.27 \text{ in.}$$

IN STEEL

$$A_{s_{comp}} = \frac{895-600}{2(50)} = 2.95 \text{ in.}^2$$

assume PNA in flange

$$x = \frac{2.95}{10.0} = 0.295 \text{ in.} < t_f = 0.645$$

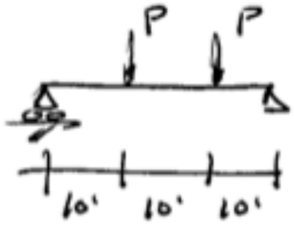



$$M_n = 895\left(\frac{13.9}{2}\right) + 600\left(4 - \frac{3.27}{2}\right) - 295\left(\frac{0.295}{2}\right)$$

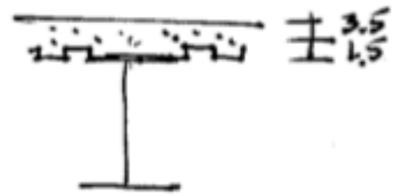
$$= 7600 \text{ in-k} \Rightarrow 633 \text{ ft-k}$$

a) $\phi M_n = 0.9(633) = \underline{\underline{570}}$ ft-k

**9-30**




$P_L = 20^k$, $P_D = 12^k$




$b_{eff} = \frac{30}{4} = 7.5 ft$
$= 90$ in.

a) LRFD

$P_u = 1.2(12) + 1.6(20) = 46.4^k$

$M_u = 46.4(10') = 464$ ft-k

assume $a = 1.0$ in $Y2 = 5 - \frac{1}{2} = 4.5$ in.

Try W16x40 PNA #3 $\Sigma Q_n = 413^k$ $\phi M_n = 492$ ft-k

$$a = \frac{413}{0.85(4)(90)} = 1.35 > 1.0$$ $\therefore$ consider $Y2 = 4.0$ in. and PNA #3

$\phi M_n = 476$ ft-k $> 464$ ft-k

Thus this will work

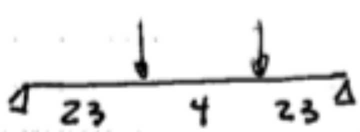
DETERMINE STUD STRENGTH

$\frac{w_r}{h_r} = \frac{2}{1.5} = 1.33 < 1.5$ $Q_n = 18.3^k$ TABLE 3-21

$$\# \text{ STUDS} = \frac{413}{18.3} = 22.6$$ $\therefore$ 23 STUDS from load point to support.

STUDS required between load points based on 36 in max or $8t = 8(5) = 40$ in.

Thus use 4 STUDS BETWEEN loads.




W16x40

STUDS

**II.** Also, answer the following problem:

Beam Design Parameters:

- Span, L = 35' (continuous lateral support)
- trib. width = 10'
- $F_y$ = 50 ksi
- Slab: $t_{slab}$ = 5" ($t_{deck}$ = 2" + $t_{LWC}$ = 3"), $w_c$ = 115 pcf, $f'_c$ = 4 ksi
- Loading:
  - $w_{slab}$ = 51 psf
  - $w_{construction}$ = 20 psf
  - $w_{partitions}$ = 20 psf
  - $w_{dead, misc.}$ = 10 psf
  - $w_{live}$ = 80 psf

**a)** Non-composite beam (steel only):

$$w_D = (51 + 10)(10) = 610 \text{ lb/ft} \text{ (neglect beam wt. and check it later.)}$$

$$w_L = (80 + 20)(10) = 1000 \text{ lb/ft}$$

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.610) + 1.6(1.000) = 2.332 \text{ lb/ft}$$

$$M_u = \frac{1}{8}w_uL^2 = \frac{1}{8}(2.332)(35)^2 = 357 \text{ ft-kips}$$

Try a W21 × 48, $\phi_bM_n = 398 \text{ ft-kips} > 357 \text{ ft-kips}$ (OK)

(continuous lateral support)

Check beam weight:

$$M_u = 357 + \frac{1}{8}(1.2 \times 0.048)(35)^2 = 366 \text{ ft-kips} < 398 \text{ ft-kips} \quad \text{(OK)}$$

Check shear: From the $Z_x$ table, $\phi_vV_n = 216$ kips

$$V_u \approx \frac{2.332(35)}{2} = 40.8 \text{ kips} < 216 \text{ kips} \quad \text{(OK)}$$

Use a W21 × 48

Compute total deflection: $w = 610 + 48 + 1000 = 1658$ lb/ft.

From the dimensions and properties table, $I_x = 959 \text{ in.}^4$

$$\Delta = \frac{5wL^4}{384EI_x} = \frac{5(1.658/12)(35 \times 12)^4}{384(29000)(959)} = 2.01 \text{ in.}$$

$\Delta = 2.01$ in. > 1.75" N.G.

L/240 = 35' x 12"/1'/240 = 1.75"

$I_{req'd}$ = 2.01/1.75(959 in$^4$) = 1102 in$^4$ : USE W21x55 I = 1140 in$^4$

RE-CHECK Phi-Vn & Phi-Mn

**b)** Composite beam (concrete slab + steel beam):

Effective flange width = $(35 \times 12)/4 = 105$ in. or $10(12) = 120$ in., use $b = 105''$

Total load to be supported by the composite section (neglecting beam weight): from Part (a),

$$M_u = \frac{1}{8}w_u L^2 = \frac{1}{8}(2.332)(35)^2 = 357 \text{ ft-kips}$$

Assume $a = 1$ in.: $\quad Y2 = t - \frac{a}{2} = 5 - \frac{1}{2} = 4.5$ in.

Try a W16 × 31. For PNA location 4, $\sum Q_n = 274$ kips, $\phi_b M_n = 362$ ft-kips

$$a = \frac{\sum Q_n}{0.85 f_c{}' b} = \frac{274}{0.85(4)(105)} = 0.7675 \text{ in.}$$

$$Y2 = t - \frac{a}{2} = 5 - \frac{0.7675}{2} = 4.616 \text{ in.}$$

By interpolation, $\phi_b M_n = 365$ ft-kips $> 357$ ft-kips (OK)

Adjust for beam weight: $\quad w_u = 2.332 + 1.2(0.031) = 2.369$ kips/ft

$$M_u = \frac{1}{8}w_u L^2 = \frac{1}{8}(2.369)(35)^2 = 363 \text{ ft-kips} < 365 \text{ ft-kips} \quad \text{(OK)}$$

Check shear: From the $Z_x$ table, $\phi_v V_n = 131$ kips

$$V_u = \frac{2.369(35)}{2} = 41.5 \text{ kips} < 131 \text{ kips} \quad \text{(OK)}$$

Before concrete cures:

$$w_D = 51(10) + 31 = 541 \text{ lb/ft}, \qquad w_L = 20(10) = 200 \text{ lb/ft}$$

$$w_u = 1.2w_D + 1.6w_L = 1.2(0.541) + 1.6(0.200) = 0.9692 \text{ k/ft}$$

$$M_u = \frac{1}{8}(0.9692)(35)^2 = 148 \text{ ft-kips}$$

From Table 3-19, $\phi_b M_p = 203$ ft-kips $> 148$ ft-kips (OK) Use a W16 × 31

Stud anchors:

Maximum stud diameter = $2.5t_f = 2.5(0.440) = 1.1$ in.

But maximum diameter with formed steel deck = $\frac{3}{4}$ in. (controls)

Minimum height of stud above top of deck = $1\frac{1}{2}$ in. Use $h_s = 2 + 1\frac{1}{2} = 3\frac{1}{2}$ in.

Try $\frac{3}{4}$-in. × $3\frac{1}{2}$-in. studs. Assume one stud at each beam location. For lightweight concrete and $f_c' = 4$ ksi, $Q_n = 17.2$ kips (Manual Table 3-21)

$$N_1 = \frac{\sum Q_n}{Q_n} = \frac{274}{17.2} = 15.9, \text{ use } 16 \text{ (32 per beam)}$$

(Actual $\sum Q_n = 16(17.2) = 275$ kips)

The approximate spacing is $\frac{L}{\text{no. studs}} = \frac{35(12)}{32} = 13.1$ in. (the exact spacing will depend on the the deck rib spacing).

Min. longitudinal spacing = $6d = 6(3/4) = 4.5$ in.

Max. longitudinal spacing = $8t = 8(5) = 40$ in. (upper limit = 36 in.)

$\therefore$ 32 studs OK. Use 32 studs, $\frac{3}{4}$-in. × $3\frac{1}{2}$-in

Deflections:

Compute total deflection.

Before concrete cures,

$$\Delta_D = \frac{5w_D L^4}{384EI_s} = \frac{5(0.541/12)(35 \times 12)^4}{384(29,000)(375)} = 1.680 \text{ in.}$$ --> 0.80 x 1.68" = 1.34" --> use 1.25" camber

$$\Delta_{const} = \frac{5w_{const} L^4}{384EI_s} = \frac{5(0.200/12)(35 \times 12)^4}{384(29,000)(375)} = 0.6209 \text{ in.}$$

Maximum deflection before concrete cures is $\Delta_D + \Delta_{const} = 1.680 + 0.6209 = 2.30$ in.

Since 1.25" camber is used, the net maximum deflection before concrete cures will be: 2.30" - 1.25" = 1.05" < L/240 O.K.

After concrete cures:

Loads applied after concrete cures:

$$w = w_L + w_{part} + w_{misc} = (80 + 20 + 10)(10) = 1100 \text{ lb/ft}$$

From Manual Table 3-20, for PNA 4 and Y2 = 4.616 in., $I_{LB} = 916.0$ in.$^4$

After the concrete cures, $$\Delta_{L+P+misc} = \frac{5wL^4}{384EI_{LB}} = \frac{5(1.100/12)(35 \times 12)^4}{384(29000)(916.0)} = 1.398 \text{ in.}$$

Maximum total deflection = $\Delta_D + \Delta_{L+P+misc} = 1.680 + 1.398 =$ $\underline{3.08 \text{ in.}}$

Since 1.25" camber is used, the net maximum deflection before concrete cures will be: 3.08" - 1.25" = 1.83" > L/240 Choose the next size up.

NOTE: when you choose a different section (i.e. W16x36), make sure you calculate the dead load deflection again, and re-calculate camber. The 1.25" camber might be too much for W16x36.